CHAPTER 6

DESIGNING STABLE FOUNDATIONS

6.1 INTRODUCTION

The design of every foundation presents a unique challenge to the engineer. Designs must take into account the nature and cost of the structure, geology and terrain, quality of subsurface investigation, kind of agreement with the owner, loadings over the life of the structure, effects of the proposed construction on buildings near the construction site, sensitivity of the proposed structure to total and differential settlement, requisite building codes, potential environmental effects due to the proposed construction including excessive noise, adequacy of the analytical tools available for making the design, result of a failure in terms of monetary loss or loss of life, availability of materials for foundations, and competent contractors in the area. A considerable effort will normally be necessary to gather and analyze the large amount of relevant data.

To create foundations that perform properly, the engineer must address carefully the topics in the above list and other relevant factors. Failures will occur if the foundation is not constructed according to plan and in a timely manner, if detrimental settlement occurs at any time after construction, if adjacent buildings are damaged, or if the design is wasteful.

The topic of designing wasteful foundations is of interest. An engineer could cause more than necessary resources to be expended on a foundation in the absence of peer review. An engineer increases the size of a foundation because the extra material gives a measure of ''pillow comfort.'' An engineer in a firm was asked to describe the technique being used to design the piles for an important job and replied: ''We use an approximate method and always add a few piles to the design.'' Such failures may never be seen, but they are failures nevertheless.

Casagrande (1964) wrote of *calculated risk* in earthwork and foundation engineering and gave several examples of projects where risk was used in the design and construction of major projects. Risk is always involved in geotechnical engineering because of the inability to get good information about the factors listed above and because of that sometimes unusual behavior of soil that cannot be anticipated in spite of diligence. Risk is minimized by prudent engineers who use available tools and techniques and who stay abreast of technological advances.

Peck (1967) presented a keynote address at a conference where the emphasis was on improved analytical procedures. He gave five sources of error with regard to the bearing capacity and settlement of foundations: (1) the assumed loading may be incorrect; (2) the soil conditions used in the design may differ from the actual soil conditions; (3) the theory used in the design may be wrong or may not apply; (4) the supporting structure may be more or less tolerant to differential movement; and (5) defects may occur during construction. The technical literature is replete with examples of foundations that have failed, many for reasons noted by Peck. The following sections present brief discussions of some examples, emphasizing the care to be taken by the engineer in planning, designing, and specifying methods of construction of foundations.

Lacy and Moskowitz (1993) made the following suggestions concerning deep foundations: understand the subsurface conditions in detail; select a qualified contractor; take care in preparing specifications for construction; provide full-time inspection by a knowledgeable person who has the necessary authority; and monitor adjacent structures as construction progresses.

6.2 TOTAL AND DIFFERENTIAL SETTLEMENT

The total settlement of a structure is the maximum amount the structure has settled with respect to its original position. The Palace of Fine Arts in Mexico has settled several feet but still remains usable because the differential settlement is tolerable. Differential settlement causes distortions in a structure, possible cracks in brittle materials, and discomfort to the occupants. The masonry building materials used in constructing the Palace of Fine Arts can tolerate the inevitable differential settlement, but some discomfort is inevitable. One can feel a definite tilting while sitting in a seat and watching a performance.

The subsequent chapters on computation of the settlement of a foundation show that the maximum settlement should occur under the center of the foundation and the minimum under the edges. Some proposals suggest that the difference between the maximum and minimum, that is, the differential, should be some fraction of the maximum. However, in all but exceptional cases, the engineer should create a design in which total settlement is moderate and differential settlement negligible.

Settlement as a result of subsidence can be important in the design of foundations. Subsidence occurs because water or perhaps oil is removed from

the underlying formations and causes an increase in effective stress, triggering settlement that can amount to several feet. Subsidence has occurred in many cities because of dewatering. Houston, Texas, is an example. The movement of the surface usually is relatively uniform and foundation problems are minimal, but faults can occur suddenly and can pass through the foundation of a home, with serious and perhaps disastrous consequences. A less common cause of subsidence is the yielding of supports for tunnels used in mines that have been abandoned in some regions of the United States.

Areas of subsidence may be more frequent in the future as the need for water increases and possibly leads to more pumping of water from aquifers. The engineer must be aware of areas where subsidence is occurring or is expected to occur.

6.3 ALLOWABLE SETTLEMENT OF STRUCTURES

6.3.1 Tolerance of Buildings to Settlement

Recommendations have been published on total and differential settlement for various classes of buildings, starting with monumental structures and ending with temporary warehouses. Total settlement is important because connections to underground services can be broken and sidewalks can be distorted and out of place. Differential settlement leads to distortions in the building and cracking of brittle building materials such as glass and ceramics. Both total settlement and differential settlement must be considered by the engineer for each structure, and published recommendations should be avoided.

The modern approach to the tolerance of buildings to settlement is to perform analyses to predict immediate and time-related movement of the foundation, consistent with the stiffness of the superstructure. Procedures presented here, such as a deformable slab supported by bearing piles, provide guidance to a comprehensive solution. Tools are available for structural engineers to use in predicting the deformation of the superstructure, taking into account nonstructural elements such as partitions and non-load-bearing walls to conform to the time-related deformation of the foundation. Field observations are used to confirm the validity of the models, but analytical tools are at hand. Engineers and owners will be relieved of the burden of complying with prescriptive requirements.

The design of some structures may be very sensitive to differential settlement. Conferences between the structural engineer and the geotechnical engineer are useful in such cases. Extraordinary methods can be employed to limit differential settlement when necessary.

6.3.2 Exceptional Case of Settlement

The Valley of Mexico, the site of Mexico City, in some ways is a large-scale laboratory for demonstrating principles of soil mechanics. Pumping from

deep-seated aquifers over many years caused several feet of subsidence in the Valley, and stresses from the foundations of major structures caused additional settlement due to the compressible nature of the soils. Zeevaert (1972) presents a profile to a depth of 80 m of the highly stratified subsurface soils in the Valley, with many meters having water contents averaging 200% and some strata with water contents as high as 400%. Settlement due to soil consolidation can be significant. Settlement in Mexico City is illustrated by the view of a government building (Figure 6.1). The building remained in use even though the settlement was much more than what might be tolerated at a site with less extreme soil conditions.

Figure 6.1 Photograph showing settlement of a government building in Mexico City.

Cummings (1941) noted that "There is no question but that Mexico City is one of the most interesting places in the world for field studies in the settlement of structures. Whereas it is often necessary to search carefully for structural settlements with precise leveling and other accurate methods, the observation of structural settlements in Mexico City can be done by the eye without the use of measuring instruments of any kind."

6.3.3 Problems in Proving Settlement

One of the authors was asked to express opinions on two sites where the failure of foundations was questionable. Structural steel was being erected for a high-rise structure in the southern United States when the contractor noticed that the steel members were not fitting together properly. Even though the settlement survey was inconclusive, the drilled-shaft foundations were underpinned without delay. More than ordinary skill is required to make precise measurements of building settlement. Even though settlement was claimed, the proof was lacking and the ensuing lawsuit was expensive, time-consuming, and perhaps unnecessary.

At a site in Hong Kong, a nearly horizontal crack was discovered in a thick pile cap during the erection of a reinforced-concrete warehouse. Settlement was postulated, even though no strong evidence was present from precise surveying. Coring of a multitude of bored piles was undertaken, and poor contact was found for some piles between their base and the founding stratum. Such piles were judged to be defective, even though the area of the core was only a fraction of the total area of the base of the pile. Hand-dug piles were being installed in some places as part of a huge program of underpinning when the decision was made to test some of the piles below the cracked cap. A slot was cut between a bored pile and the cap, flat jacks were installed, the height of the existing reinforced-concrete structure provided sufficient resistance, and load testing proceeded. Three piles were tested with the flat-jack method, and all showed ample capacity. Underpinning was discontinued, and the construction proceeded without a problem. Later, an investigation revealed that the crack in the pile cap was due to a cold joint resulting from a large concrete pour.

6.4 SOIL INVESTIGATIONS APPROPRIATE TO DESIGN

6.4.1 Planning

Many factors affect the plans for a proper investigation of the subsurface soils for a project. A cooperative effort is desirable in which the owner conveys to the architect and structural engineer the requirements for the proposed structure, the structural engineer and architect make a preliminary plan that dictates the foundation loads, and the geotechnical engineer describes the geology and

suggests a type of foundation. Unfortunately, the geotechnical engineer is often selected later, after a plan is proposed for the subsurface investigation where price could be an important consideration. A cursory study of the soils could result, with the geotechnical engineer recommending design parameters on the basis of limited information. Such limited soil studies have resulted in a large number of claims by the contractor of a ''change of conditions'' when the soils were not as presented in the soils report.

The ideal plan, unless substantial information on subsurface conditions is available, is to perform exploratory borings for classification and to get data that will guide the borings for design. As noted in Chapter 4, a wide range of techniques are available to the geotechnical engineer. The data will allow the type of foundation to be selected and will provide numerical values for the relevant soil properties.

6.4.2 Favorable Profiles

At a number of locations in the United States and elsewhere, the use of bearing piles is dictated by the nature of the soil overlying the founding stratum. The soil investigation is aimed mainly at determining the thickness of the surface stratum in order to find the required length of the piles. Penetration tests can be used with confidence, and experience may show that the piles need only be driven to refusal into the bearing stratum.

For many low-rise buildings supported on a thick surface stratum of sand, the SPT (Chapter 4) can be used to determine that the sand is not loose or very loose and to ascertain the position of the water table. Spread footings or a raft can be designed with confidence.

Other soil profiles exist that are well known to local engineers, and the type of foundation can be selected without exploratory borings. The soil investigation can be made with methods that are locally acceptable and lead to a standard design. The geotechnical engineer must exercise caution in all cases to identify soils with special characteristics as discussed below. Nature is anything but predictable, and the engineer must be alert when characterizing soils.

6.4.3 Soils with Special Characteristics

Cambefort (1965) wrote of experiences with unusual soils and remarked: ''There is no known 'recipe.' But the most dangerous thing is to think that the known formulas can explain everything. Only reasoned observation can lead to satisfactory results.'' The observational method should always be used, particularly on major projects, and some engineers have employed this method extensively. The thesis is that not all the features of the behavior of a foundation can be predicted but that strengthening can be done if excessive movement is observed.

The following sections describe several of types of soil that can cause problems in construction and deserve special attention from the engineer. Other sections of the book present methods of designing foundations for a number of soils noted below.

6.4.4 Calcareous Soil

The engineer may encounter an unusual soil, especially if working in an area where little is known about the soil. Several years ago, one of the authors attended a preconstruction meeting prior to building an offshore platform on the Northwest Shelf of Australia. A sample of sand was shown, and the results of laboratory testing were presented. The friction angle was consistent with the relative density, and the decision was made to design the piles using available equations even though the sand was calcareous due to the nature of the geologic deposition.

The template was set on the ocean floor, and piles were to be stabbed and driven with the legs of the template as guides (Jewell and Andrews, 1988). However, after the first open-ended pipe pile was placed into the template, the pile fell suddenly about 100 m and came to rest on a dense stratum. The calcareous grains had crushed under the walls of the pile, and natural cementation in the deposit prevented the calcareous sand from exerting any significant lateral stress against the walls of the pile. A stress-strain curve from the laboratory exhibited severe strain softening. Thus, skin friction was low to nonexistent as a result of the large deformation during installation, and the allowable end bearing on the piles was insufficient to provide adequate safety during the design storm. The result was that special strengthening was designed after an intensive study. The required construction was expensive, complex, and time-consuming.

Very Soft Clay The design of stable foundations must address the presence of soft clay at the construction site. Several of the chapters of this book deals with aspects of soft clay, including identification, strength, deformational characteristics, and design of foundations. Two procedures are noted below. In addition, the engineer must take special care if an excavation must be made in soft clay. Analytical techniques must be employed to investigate the possibility of sliding that would affect the site and possibly nearby structures as well.

Preloading. If the construction can be delayed for a period of time, the site may be preloaded with a temporary fill. A drainage layer can be placed on the surface of the soft clay, and boring can be used to install vertical drains at appropriate spacing. Analysis can predict the time required for the clay to drain to an appropriate amount, with a consequent increase in shear strength. Settlement plates can be installed to provide data to confirm the analytical predictions or to allow modification of the predictions.

Load-Bearing Piles. Another method of providing foundations where there is a stratum of very soft clay is to install piles through the clay to a bearing stratum below. Usually piles are driven, causing modification of the properties of the clay. The engineer must be aware that soft clay can settle, subjecting the piles to downdrag. Possible problems related to the buckling of piles in soft clay can be investigated by methods presented in Chapters 12 and 14.

Expansive Clay Expansive clay at a construction site, if not recognized, can sometimes lead to disastrous results. Chapters 2, 3, 6, and 9 will discuss the identification of expansive clay and the design of shallow and deep foundations. Expansive clay increases the cost of the foundation for a low-rise structure, and inadequate foundations are being built in spite of current knowledge and some building codes. Thus, the problem is faced more by homeowners than by the owners of commercial buildings.

Clay expands as it becomes wet and shrinks as it dries. Furthermore, moisture will collect when evaporation is cut off. In addition to swelling of the clay, other important factors are the nature of the foundation, weather, and transmission of moisture through the clay. A foundation on expansive clay may show no distress for perhaps years and then experience severe movement. On the other hand, one of the authors was asked to visit a site where a church building had cracks in the walls so wide that people in the congregation could see children in a playground. However, a rain had occurred the night before, and when the author arrived, the cracks were virtually closed! Figure 6.2 shows the cracking of a structure on expansive clay.

The severe differential movement of the foundations of homes on expansive clay can sometimes be devastating. In a home belonging to an assistant sports coach at a major university, doors did not close properly, the wallpaper was wrinkled, and the floors were very uneven. A portion of the home was on a slab, and another portion was supported by piers and beams. Repairs could be made, but the expense would be heavy. Later, the coach was divorced, and he moved away.

Some years ago, a nonprofit agency was asked to host a series of seminars on expansive clay. The plan was to encourage potential homeowners to look for cracks in the ground, damage to nearby homes, and elementary methods of identifying expansive clay. Geologists, geotechnical engineers, structural engineers, developers, and local officials were to be invited, along with the potential homeowners. The agency declined to host the seminars, perhaps due to fear of being sued by builders or developers who had tracts where expansive clay existed.

In addition to presenting information on identification of expansive clay, this book will give recommendations for the construction of shallow and deep foundations.

Loess In the Mississippi Valley of the United States, in Romania, in Russia, and in many other parts of the world, a soil exists called *loess*. It was created

Figure 6.2 Damage to a masonry structure on expansive clay.

by the transport by wind over long periods of fine grains ranging in size from about 0.01 to 0.05 mm. Grass or other vegetation grew during the deposition, so loess has a pronounced vertical structure with cementation associated with the vegetation.

Cuts in loess will stand almost vertically to considerable heights, but the soil will collapse under load when saturated. Loess is capable of sustaining a considerable load from a spread footing, but the design of foundations must consider the possibility of saturation. For relatively light structures, loess may be treated to a depth of up to 2 m by prewetting, compaction, and/or chemicals. Appropriate drainage is critical. Pile foundations extending through the loess are frequently recommended for major structures.

Loose Sand Terzaghi (1951) described the design of foundations for a factory building in Denver, Colorado, where the assumption was made of an *allowable bearing value* of 2 tons/ft^2 for the underlying sand. The dead load from the building was 0.9 ton/ft^2, but when a heavy snowfall increased the loading to 1.4 tons/ft^2, the building experienced settlement of the columns of up to 3.5 in. Tests performed later showed the sand to be loose to very loose and variable both vertically and horizontally.

A serious problem with loose sand is densification due to vibration. Vibration will cause the void ratio to decrease and settlement to occur. Problems

have been reported with foundations of pumps at pipelines in South Texas, where unequal settlement occurred after operation for some time.

Pinnacle Limestone and Embedded Boulders Pinnacle limestone, where a deposit of limestone is riddled with solution cavities, and embedded boulders present similar problems. Both kinds of sites are extremely difficult to investigate by subsurface drilling or probing.

Pinnacle limestone is prevalent in the southeastern United States and elsewhere and is generally known by the geology of the area. Each site poses a different problem, and no straightforward method of determining an appropriate foundation is evident. Drilled shafts are usually recommended, with the depth of the foundation depending on the result of drilling.

A vertical surface exhibiting pinnacle limestone at the site of the foundation for the Bill Emerson Bridge in Missouri is shown in Figure 6.3. The existence of solution cavities is apparent. About 360 boreholes were made at the site of one of the foundations with a surface area of 90 by 120 ft. The subsurface condition was revealed in a three-dimensional plot generated by a computer. The computer depiction allowed slices to be taken through the formation, and a program of grouting was undertaken to eliminate zones of weakness. A foundation of drilled shafts was then executed.

Extraordinary solutions are sometimes required in constructing the foundations when pinnacle limestone exists at the site. At a construction site in Birmingham, Alabama, with dimensions of about 200 by 280 ft, three types of deep foundations were required: drilled shafts for about 40% of the site, micropiles for about 27%, and driven pipe piles for the remaining 33%. One

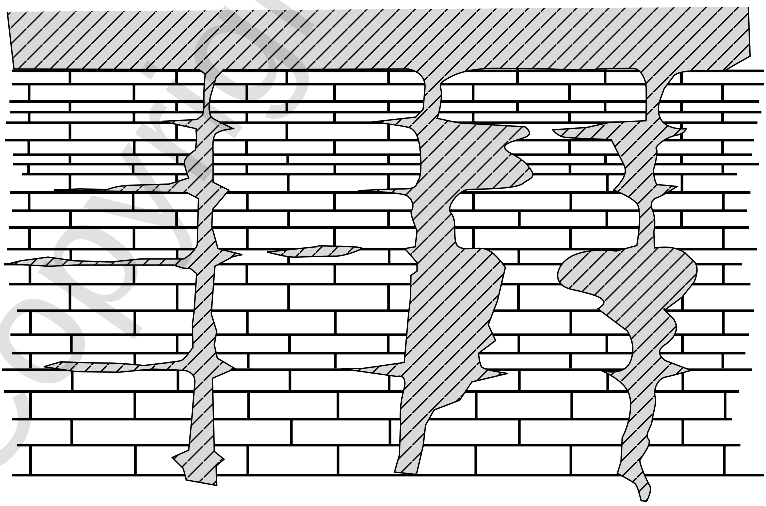

Figure 6.3 Condition of limestone at the site of the Bill Emerson Bridge, Missouri (from Miller, 2003).

of the micropiles extended to a penetration of 37 ft, and a few feet away another extended to a depth of 129 ft. The driven piles are anticipated to penetrate to a depth of 150 ft. The design created challenges for the engineer and the contractor to ensure that the axial movements of the different types of piles would be within the allowable range for the superstructure.

Embedded boulders create severe difficulties in designing and constructing deep foundations. Boulders in weaker soil occur because of the action of glaciers, or because of uneven weathering, or possibly because of being left in fills in the past. D'Appolonia and Spanovich (1964) describe large settlements of an ore dock because of the different response of supporting piles in settlement under axial load. Some of the 6000 piles rested on boulders and others were founded in hardpan. The authors stated that the boulder-supported piles were more compressible because of the short-term settlement of the soil beneath a boulder, causing load to be transferred to piles founded in hardpan, which then became overloaded.

If boulders are known to exist at a site, the design may call for the use of drilled shafts (bored piles). If the boulders are smaller than the diameter of the drilled hole, "grab" tools can be used to lift the boulders from the excavation and drilling can proceed. If the boulders are larger than the diameter of the drilled hole, special techniques are required. The boulders may consist of soft rock and can be broken by the use of a chopping bit; if they consist of hard rock, the size of the drilled hole may be increased. Sometimes the hard rock of the boulder can be drilled and a steel bolt can be grouted into place to allow the boulder to be lifted by a crane. Extreme care must be used if workmen enter a drilled hole because of the danger that carbon monoxide gas has settled into the excavation from a nearby motorway.

6.5 USE OF VALID ANALYTICAL METHODS

Models of various kinds have been proposed for the solution of every foundation problem. For example, failure surfaces are shown in the soil below a footing, and soil-mechanics theory is used to predict the location and stresses along these surfaces. Integration is then used to compute the load on the footing that generates the failure surfaces. Such a model is difficult to apply when obtaining the bearing capacity of a footing on layered soils, particularly considering three-dimensional behavior. But such models can be used with confidence if proven by full-scale load tests.

While the ultimate load on a footing may be computed with the model described above, another type of model must be used to obtain the movement of the foundation. The goal of analytical techniques is to have a model that may be used both for ultimate capacity and for the nonlinear movement of the foundation. The development of such models is continuing.

6.5.1 Oil Tank in Norway

Bjerrum and Överland (1957) studied the failure of an oil tank in Norway. Soil borings were made at the site of the tank. Properties of the soil were obtained by the use of the in situ vane and by the unconfined compression test. The upper layer of soil was a silty clay to a depth of 7 m with a soft marine clay below. The undrained shear strength was reconstituted to obtain values for the construction time of the tanks. The strength of the soil was almost constant to a depth of 10 m with a value of about 3 tons/m^2, but the shear strength was over 4 tons/m^2 at a depth of 15 m. The diameter of the tank was 25 m. If a failure of the entire tank was assumed, the average shear strength along the failure surface yielded a factor of safety of 1.72. On the other hand, if failure was assumed at one edge of the tank, the average shear strength along the shallower failure surface was less and the factor of safety was computed to be 1.05.

The heave of the soil was near one edge of the tank and indicated a local failure in which the weaker soil was mobilized. The error in the original design was due to the selection of a model for general failure where the average shear strength along an assumed failure surface was significantly larger than the average shear strength of the near-surface soil related to local failure. Thus, the careful selection of a model for a failure surface that will yield the lowest factor of safety is essential.

6.5.2 Transcona Elevator in Canada

This grain elevator located near Winnipeg, Canada, was constructed in 1913 on a raft foundation. Failure of the bin house by gradual tilting occurred after 875,000 bushels of wheat had been stored yielding a load of 20,000 tons, distributed uniformly. The bin house had plan dimensions of 77 by 195 ft and was 92 ft high. The elevator was founded on clay that had been deposited in a glacial lake and failed by tilting when the uniform load reached 3.06 tsf. The raft was 12 ft below the ground surface, giving a net increase in load of $3.06 - (120 \times 12) = 2.34$ tsf.

Soil borings were made in 1953, and properties were determined by testing of tube samples (Peck and Bryant, 1953). The stratum below the raft was 27 ft thick, and had an unconfined compressive strength (q_u) of 1.13 tsf and a sensitivity of 2. The next stratum was 8 ft thick, with a q_u of 0.65 tsf and a sensitivity of 2. The clay was underlain by a granular stratum with refusal to the penetration test at 20 ft below the clay.

The depth of the raft and its plan dimensions yielded a bearing capacity factor N_c of 5.56. Using a weighted average for the 35 ft of clay beneath the raft of 0.93 tsf, the allowable loading was computed to be $(0.5)(0.93)(5.56) = 2.57$ tsf. Thus, the factor of safety against collapse was slightly greater than 1. Many codes of practice, along with guidelines used by practicing

engineers, suggest using a factor of safety of up to 3 to account for errors in obtaining soil properties and limitations of the theory.

6.5.3 Bearing Piles in China

The model for the design of a pile under axial loading (see Chapter 10) is fairly simple. For transfer of load in side resistance (skin friction) the model employs the distribution of stresses on the pile–soil interface of elements along the length of the pile. The axial load sustained by an element can be computed by integrating the vertical stresses along the face of the element. For transfer of load at the base of the pile (end bearing), the model employed is similar to that for a footing. If piles are driven close to each other, an allowance must be made for pile–soil–pile interaction.

While the models described above have been in use for many years, a design in 1959 used piles of different lengths along a pier (Figure 6.4) where the axial loading on the deck of the pier was presumably uniform. The shorter piles settled more than the longer ones and caused an unacceptable and uneven settlement of the pier. Long et al. (1983) describe the repair of the pier

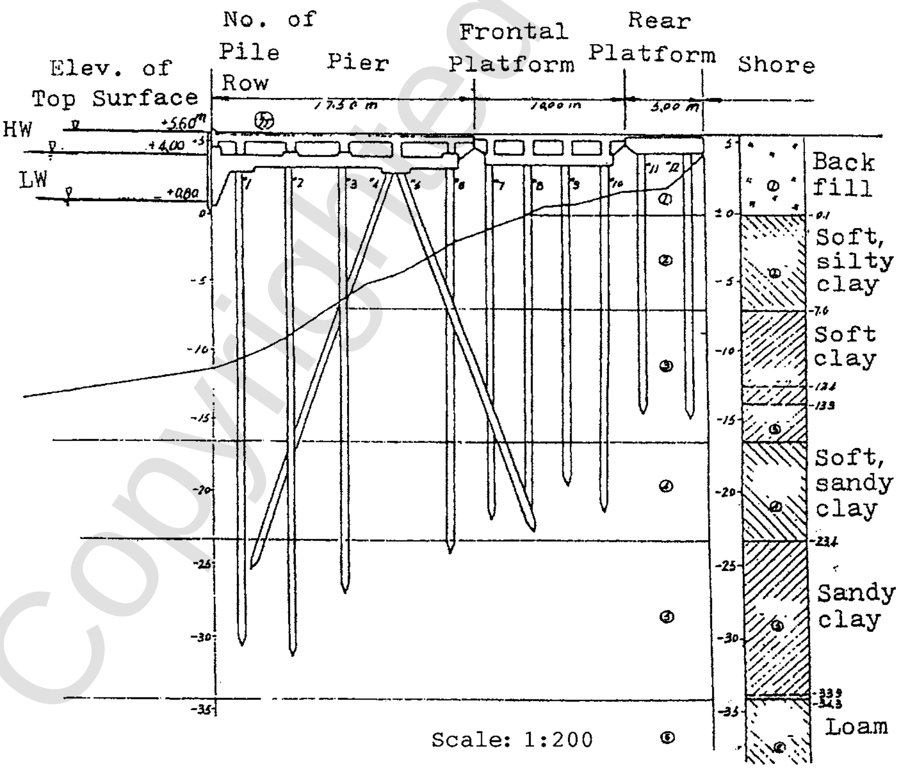

Figure 6.4 Structure of a pier and typical boring log (from Long et al., 1983).

caused by the unequal settlement of the piles. The soil profile across the site was relatively uniform. The use of the simple models described above should have produced an acceptable design. However, the use of the models for an axially loaded pile is obviously more complex if the soil profile varies across the construction site.

6.6 FOUNDATIONS AT UNSTABLE SLOPES

Chapter 3 presents an equation for the shearing resistance of soils: $s = c + (p - u) \tan \phi$, where c = cohesion, p = total normal stress on the plane of failure, u = porewater pressure, and ϕ = friction angle for the soil. The equation shows that if the value of u becomes equal to p, the shearing resistance of the soil becomes equal to c, which could be quite small for a clay or equal to zero for a sand. The value of the porewater pressure can increase as surface loading is applied, say from a building, and if the formation does not allow rapid drainage, the strength of the soil will be reduced. The following examples illustrate the importance of predicting the porewater pressure.

6.6.1 Pendleton Levee

A failure occurred as described by Fields and Wells (1944). The Pendleton Levee was built along the Mississippi River above a formation consisting of 4 to 9 ft of fine sand, underlain by 12 ft of soft clay, and underlain in turn by a relatively thick stratum of fine to coarse sand. Specimens of the clay were tested in the laboratory without drainage and were found to have a c value of 80 psf and a ϕ value of l9°. The clay exhibited strain softening with the ϕ value of 12°, corresponding to the ultimate strength of the clay.

The soil for the levee was moved and distributed by earth-moving equipment. When the fill reached the maximum height of 32 ft, a slide caused the levee to fail on the land side. The levee had been instrumented extensively to gain information on present and future construction.

Terzaghi (1944) studied the results of the soil investigation and the data from the instruments and concluded that failure had occurred in a thin horizontal stratum of fine sand or coarse silt in the soft clay that was undetected in the soil investigation. The layer of granular soil could reasonably have occurred during the process of deposition over the centuries as flooding deposited soil in layers. Variations in climatic conditions could have led to the deposition of the cohesionless soil. The fill caused the pore pressures to increase in the thin, cohesionless stratum, and failure occurred along that stratum as its strength was reduced.

Peck (1944) discussed the paper and noted that the identification of such a stratum of fine sand in the clay would require continuous sampling with proper tools. Kjellman et al. (1951) described the use of the Swedish sampler with thin strips of foil along the interior of a sampling tube. The foil is

retracted as the sampler penetrates the soil, eliminating the resistance between the soil and the interior of the tube and allowing undisturbed samples to be obtained with lengths of several meters. Such a tool would have been ideal for sampling the soil at the Pendleton Levee but the method has found little use outside of Sweden.

6.6.2 Fort Peck Dam

A slide occurred at Fort Peck Dam that allowed impounded water to rush out, with consequent damage and public outcry. A comprehensive investigation followed, and Middlebrooks (1940) reported that the slide occurred in shale with seams of bentonite at the hydraulic-fill dam. The investigation prior to design failed to indicate the extent of weathering in the shale and bentonitic seams; further, the investigation failed to predict the high hydrostatic pressure in the bentonite due to the load from the hydraulic fill.

6.7 EFFECTS OF INSTALLATION ON THE QUALITY OF DEEP FOUNDATIONS

6.7.1 Introduction

All types of deep foundations can be damaged by improper methods of construction. As noted by Lacy and Moskowitz (1993), the engineer has several responsibilities prior to and during construction. Specifications for construction must be prepared that give the methods to be used to achieve foundations of good quality. Also, it is critical for qualified inspectors to be present during construction to ensure compliance with the specifications. The selection of a qualified contractor with experience with the proposed system and with proper equipment is vital, and the owners of the project should allow procedures that ensure a qualified contractor, such as prequalification of bidders. Preconstruction meetings with the successful contractor are almost always useful.

Driven Piles Piles are sometimes damaged by overdriving. The wave-equation method is useful to select an appropriate pile hammer for the pile to be driven. Attention to detail is important concerning such things as the driving equipment, the quality of the pile being driven, and the arrangements of the components of the system. Even with the use of an appropriate hammer and appurtenances, tough piles such as H-piles can be damaged at their ends by overdriving.

Damage of steel piles in unusual ways can involve great expense and loss of time. Open-ended steel pipes were being driven to support an offshore platform when the lower ends were distorted by being banged against the template structure. The distortion caused the lower portion of the piles to buckle inward during driving due to lateral forces from the soil. The collapse

prevented drilling through the piles to allow the installation of grouted inserts. Many engineers worked for several months to design a remedial foundation.

Damage to precast concrete piles during driving can be vexing and expensive. The authors are aware of a bridge along the coast of Texas where many of the precast piles had been damaged due to overdriving, causing tensile cracks at intervals along the piles. Salt water entered the cracks, corroded the reinforcing steel, and destroyed the integrity of the piles. The contractor worked beneath the deck of the bridge to install a special system of drilled piles.

Selection and proper replacement of cushioning material when driving concrete piles is important (Womack et al., 2003). The authors were asked to review the installation of concrete piles where plywood was used as cushioning material. Driving continued until the plywood actually collapsed and burned due to continued use. The concrete piles exhibited a number of cracks in the portions above the ground.

Drilled Shafts Drilled shafts or bored piles are becoming increasingly popular for several reasons. The noise of installation is less than that of driven piles, drilling can penetrate soft rock to provide resistance in side resistance and end bearing, and diameters and penetrations are almost without limit. Drilled shafts are discussed in Chapter 11, and construction methods are described in detail in Chapter 5.

As noted earlier, preparation of detailed specifications for construction of drilled shafts is essential, and a qualified, knowledgeable engineer must provide inspection. Preconstruction meetings are highly desirable to ensure that the specified details of construction are consistent with the contractor's ability to construct the project.

CFA Piles As noted earlier, CFA piles have been used for many years and are competitive with respect to the cost per ton of load to be supported when the subsurface conditions are suitable. The CFA pile is constructed by rotating a hollow-stem CFA into the soil to the depth selected in the design. Cement grout is then injected through the hollow stem as the auger is withdrawn. Two errors in construction with CFA piles must be avoided: (1) the mining of soil when an obstruction is encountered and the auger continues to be rotated and (2) abrupt withdrawal of the auger, allowing the supporting soil to collapse, causing a neck in the pile. To assist in preventing these two construction problems, a data-acquisition system can be utilized to record rotations, grout pressure, and grout volume, all as a function of penetration of the auger. Alternatively, the inspector can obtain such data on each pile as construction progresses.

Other Types of Piles Many other types of piles for deep foundations are described in Chapter 5. The engineer must understand the function of such piles and the details of the construction methods. Preparation of appropriate

specifications for construction and inspection of construction are essential. Field load tests of full-sized piles often must be recommended if such experimental data are unavailable from sites where the subsurface conditions are similar.

6.8 EFFECTS OF INSTALLATION OF DEEP FOUNDATIONS ON NEARBY STRUCTURES

6.8.1 Driving Piles

The installation of deep foundations obviously affects the properties of the nearby soils. Such effects may be considered in design. In addition, movements of soil from installation of piles must be considered. The driving of a pile will displace an amount of soil that can affect nearby construction. The displacement is greater if a solid pile is driven, such as a reinforced-concrete section, and less if an H-pile or an open-ended-pipe pile is driven. However, in some soils, the pipe pile will plug and the soil between the flanges will move with the pile; these piles can become *displacement* piles.

When a displacement pile is driven into clay, the ground surface will move upward, or heave, and the heave can cause previously driven piles to move up. Often the heaved piles must be retapped to restore end bearing. The heave and lateral movement could also affect existing structures, depending on the distance to the structures. The volume of the heaved soil has been measured and is a percentage of the volume of the driven piles.

If a pile is driven into loose sand, the vibration will cause the sand to become denser and the ground surface will frequently settle. Settlement occurs even though a volume of soil is displaced by the placement of the pile. As noted earlier, settlement will occur if mats or spread footings on loose sand are subjected to vibratory loadings.

Lacy and Gould (1985) describe a case where fine sand and varved silt from glacial outwash overlie bouldery till at Foley Square, New York City. H-piles were driven for a high-rise structure on 3-ft centers through 80 ft of sand and silt. The vibrations from pile driving caused settlement of adjacent buildings founded on footings above the glacial sand. Even though the construction procedures were changed, the vibration of the sand at the adjacent buildings caused settlement of the footings to continue. The adjacent buildings, 6 and 16 stories in height, had to be demolished.

The driving of any type of pile will cause vibrations to be transmitted, with the magnitude of the vibrations dependent on the distance from the construction site. Predicting whether or not such driving will damage an existing structure requires careful attention. The engineer must be cognizant of the possibility of such damage and take necessary precautions. Mohan et al. (1970) present the details of damage to existing buildings in Calcutta due to nearby pile driving.

The near-surface soils in Calcutta consist of silts and clays to a depth of about 15 m. Foundations of relatively small buildings can be placed in the top 5 m, where the clay has medium stiffness. Larger buildings are founded on piles that penetrate the 15 m into stronger soil below. Piles were being driven at a site where the central portion of the site was about 30 m from two existing buildings that were founded on spread footings. After 145 reinforced-concrete piles had been driven, cracks were observed in the existing buildings. While there was no danger of failure of the spread footings, the cracks were unsightly and reduced the quality of the buildings. A study was undertaken before driving the last 96 piles for the new building. Driving at more than 30 m from the site caused no damage to existing buildings, but as the driving moved closer, the height of fall of the pile hammer had to be reduced from 1 m to 15–30 cm.

Construction of a large federal building in New York City was halted when only a third of the piles had been driven. The building had plan dimensions of 165 by 150 ft and was to be 45 stories high. It was to be supported on 2,700 14BP73 piles. The water table was 15 ft below street level, and much of the underlying soil was medium to fine sand. The subsidence due to the pile driving had lowered a nearby street more than 12 in., and sewer lines had been broken on two occasions (*ENR,* 1963b, p. 20).

Lacy et al. (1994) recommend the use of CFA piles to reduce the impact on adjacent structures—for example, by eliminating densification (''loss of ground'') of granular soil as a result of pile driving.

6.9 EFFECTS OF EXCAVATIONS ON NEARBY STRUCTURES

Excavation near existing structures can cause two problems if the excavation is carried out below the water table. First, the cut, either open or braced, can allow the soil to move toward the excavation, resulting in lateral movement of the foundations of an existing structure. Second, the lowering of the water table will result in the possible drop of the water table at some distance away from the cut. The increase in the effective stress due to the lowered water table can cause settlement in some soils.

With respect to the latter case, an excavation was made in Renton, Washington, for a sewer line. The excavation traversed an old river channel and an aquifer that existed below the bottom of the trench. Dewatering was done with large-diameter deep-well pumps. The Boeing Company claimed that the dewatering of the excavation caused significant damage to a pile-supported two-story steel-frame structure built at a site that was formerly a peat bog. Even though the building was 1,300 ft away from the dewatered excavation, settling of the ground floor was observed and cracks appeared in the interior walls (*ENR,* 1962).

6.10 DELETERIOUS EFFECTS OF THE ENVIRONMENT ON FOUNDATIONS

Steel pipes exposed to a corrosive environment can be damaged severely. Designs must address two conditions: corrosive water in natural soil deposits and structures in sea water. A soil investigation must determine the character of the water. If the water is found to be corrosive, the engineer may provide extra wall thickness to allow for an amount of loss of metal throughout the life of the structure or may provide a coating for the piles. Several types of coatings or wraps may be used but, in any case, the engineer must be assured by preparing appropriate specifications that the installation of the piles does not damage the coating or wrap.

Steel piles that support waterfront or offshore structures in the oceans must be protected against corrosion by the use of extra metal in the splash zone or by coatings. Alternatively, the engineer may specify the employment of cathodic protection, in which sacrificial metal ingots are installed in connection with appropriate electrical circuits.

6.11 SCOUR OF SOIL AT FOUNDATIONS

The scour or erosion of soils along streams or at offshore locations can be catastrophic. If soil is eroded around the piles supporting bridge bents, the possible collapse of the structure will cause inconvenience, may be very expensive, and could result in loss of life. Unfortunately, technical literature contains many examples of such failures (*ENR,* 1962).

Predicting the amount of scour is a complex undertaking. Predictions can be made of the velocity of a stream to put in suspension a particle of granular soil, up to the velocity of a mountain stream that moves boulders. Moore and Masch (1962) developed a laboratory apparatus that could be used to study the scour of cohesive soil. It is vital to predict the increase in velocity when an obstruction, such as a foundation pile, is in the stream.

Simply placing large stones around the foundations to prevent scour will not always suffice because in time the stones can settle into the fine soil beneath. One solution is to employ a reverse filter, in which layers of granular soil of increasing size are placed on the fine natural soil so that the particles at the surface are sufficiently large to remain in place during swift stream flow. The design of the reverse filter follows recommendations of Terzaghi on research at the Waterways Experiment Station at Vicksburg, the so-called TV grading (Posey, 1971).

PROBLEMS

P6.1. Develop a table identifying the failures mentioned in this chapter and state whether the incident (a) caused a minor monetary loss, (b) caused

a major monetary loss, or (c) could have been a catastrophe in which loss of life was possible. Note: a particular failure could have involved more than one category.

P6.2. Using one of the cases in Problem P.6.1, write a short statement giving your opinion on the reasons for the noted failure. Some of the reasons may be an inadequate soil investigation, including inadequate interpretation; lack of the proper analytical procedure; failure to use the available analysis; failure in the construction operations; or other reasons.